Chapter 18

Seismic resistance of a reinforced concrete building retrofitted via base isolation

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Abstract

This paper deals with the evaluation of the seismic resistance of a reinforced concrete building that originally was not designed to resist earthquake loading and afterwards was subjected to a seismic retrofitting that included both a partial reinforcement of the existing structure and a base isolation system. The seismic performance of the retrofitted building has been evaluated with reference to the collapse conditions. To this aim 3-D nonlinear static analyses have been performed and a procedure for the definition of simplified two-degrees-of-freedom nonlinear systems, equivalent to the base isolated building, has been assessed. The equivalent simplified systems allowed us to perform a wide number of nonlinear dynamical analyses with reference to several seismic inputs characterized by different levels of peak ground acceleration. The ultimate capacity of the retrofitted building has been conventionally expressed in terms of the minimum PGA that causes the collapse of the building according to a prescribed design spectral shape.

1 Introduction

The effects of earthquakes on existing seismic resistant structures has shown that the design philosophy based on ductility must be critically reviewed because the recourse to ductility has caused significant structural damage and, as a consequence, very high economic losses [1]. Therefore an innovative approach based on passive energy dissipation systems and base isolation systems is
certainly preferable to the classical approach based on structural strengthening and ductility. These novel techniques are nowadays widely used in seismic areas and several examples of passive controlled structures have already been constructed all over the world. In many cases this innovative approach has been successfully applied for the seismic retrofitting of existing structures for which the study of seismic vulnerability and seismic resistance is preliminary to the subsequent seismic rehabilitation. This work deals with the evaluation of the seismic resistance of a retrofitted reinforced concrete building, that originally was not designed to resist earthquake loading and whose seismic resistance, before the retrofitting, has been estimated in a companion paper [2]. For this case study building a seismic rehabilitation, that includes both a partial reinforcement of the existing structure and a base isolation system, has been designed and currently is being carried out [3]. The seismic rehabilitation of the superstructure is provided by means of thin shear walls, while the base isolation system is constituted of laminated rubber bearings. In the 3-D finite element model the behaviour of the shear walls has been modelled according to discrete element models [4, 5, 6, 7, 8], while the isolator devices have been introduced by means of nonlinear spring-dashpot elements. The seismic resistance of the retrofitted building, with reference to the expected earthquake action for the construction site, is estimated by means of procedures based on the results of pushover analyses on 3-D models as well as dynamic analyses on two degrees simplified equivalent systems. These simplified systems allow an accurate modelling of the nonlinear stiffness and damping properties of the isolator devices, permitting us to perform a wide number of nonlinear dynamical analyses that produces an estimation of the seismic resistance of the retrofitted building with a strongly reduced computational effort. The seismic performance of the retrofitted building has been evaluated with reference to the collapse conditions and it has been conventionally expressed in terms of the minimum PGA that causes the collapse of the building according to a prescribed design spectral shape. The obtained results show a very good performance of the isolated structure that is able to withstand the expected earthquake on the construction site without suffering any damage.

2 The seismic retrofitting project

The seismic retrofitting of the building has been designed by a team of structural engineers [3]. The retrofitting project includes both a partial reinforcement of the existing structure and a base isolation of the building. The use of a base isolation system in an existing structure was, to some extent, suggested by the good characteristics of the subsoil and by the presence of squat columns, just above the foundation, that facilitated the insertion of the isolators as can be seen in Fig. 1.

The reinforcement of the superstructure was essential due to the poor characteristics of the concrete in the existing structure. In the following a brief description of the retrofitting project is provided.
Figure 1: Squat columns in the original building (a) before and (b) after the insertion of the seismic isolators.

![Figure 1](image)

Figure 2: Plan of the foundation structure with the base isolation system.

![Figure 2](image)

**2.1 Base isolation**

In Fig. 2 the plan of the foundation with the location of the base isolation devices is reported. The isolation system consists of twelve laminated rubber bearing devices and of low friction sliding supports. Both the isolators and the sliders are located beneath the columns. The nonlinear characteristics of the seismic isolation devices are represented in Figs. 3; namely in Fig. 3a the variation of the damping ratio as a function of the lateral displacement is reported, while in Fig. 3b the nonlinear force/displacement relationship is represented. The main characteristics of the low-stiffness isolation devices are summarized in Table 1.

**2.2 Strengthening of the foundation and shear walls**

With reference to the existing structure, besides the consolidation of the foundation, the retrofitting project provides strengthening of the superstructure.
by means of thin reinforced concrete walls. In Fig. 4 the typical plan of the reinforced structure with an indication of the new structural elements is reported.

Table 1: Main characteristics of the laminated rubber isolation devices.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate lateral displacement</td>
<td>141 mm</td>
</tr>
<tr>
<td>Lateral stiffness at the ultimate lateral displacement</td>
<td>0.77 kN/mm</td>
</tr>
<tr>
<td>Design lateral displacement</td>
<td>13 mm</td>
</tr>
<tr>
<td>Lateral stiffness at the design lateral displacement</td>
<td>7.7 kN/mm</td>
</tr>
<tr>
<td>Steel plate diameter and thickness</td>
<td>∅ 490 mm / 3 mm</td>
</tr>
<tr>
<td>Total rubber thickness</td>
<td>94 mm</td>
</tr>
</tbody>
</table>

Figure 3: (a) Variation of the damping ratio as a function of the lateral deformation and (b) lateral force/displacement relationship for the isolation devices.

Figure 4: Typical floor plan with an indication of the new thin shear walls.
3 Discrete models of shear walls

In order to reduce the computational effort, in the 3-D finite element model the reinforced concrete structural walls have been modelled by means of discrete equivalent models. These are able to describe the nonlinear behaviour of bi-dimensional elements through equivalent mechanical schemes with lumped plasticity. Namely the Multi-Component-in-Parallel (MCP) model, introduced by Vulcano and Bertero [6, 7, 8], has been adopted. According to this model, a single concrete wall can be schematized by means of $2n$ vertical plus one horizontal nonlinear springs which are connected by rigid elements as reported in Fig. 5. The vertical springs mainly describe the axial and the flexural behaviour, while the horizontal spring essentially simulates the shear behaviour of the wall.

3.1 Linear characteristics of the MCP model

In the 3-D nonlinear model the rigid elements, which are able to move one with respect to the other causing the reaction of the nonlinear springs, are rigidly connected to the floor levels. Each MCP element must be set both in the elastic and inelastic state. According to the original model [6, 7, 8] empirical relationships need to be used for the evaluation of the main parameters that characterize both the elastic and the inelastic behaviour of the model. In the present work an alternative criterion that allows a better estimation of the initial elastic parameters of each element has been introduced. This is simply based on the equivalence between the flexibility matrix of the MCP element and that of a 2-D elastic shell elements model of the structural wall. In both the models the wall is fully restrained at the base; furthermore in the 2-D model a rigid constraint is imposed to the upper nodes (Fig. 6).

Both the discrete and the finite element models own three degrees-of-freedom that can be characterized through the two displacement components $u$, horizontal, and $v$, vertical, and the rotation $\varphi$ of the upper rigid element (Fig. 6).

![Figure 5: The Multi-Component-in-Parallel (MCP) model.](image-url)
Figure 6: Lagrangian parameters and corresponding forces for (a) the MCP and (b) the shell elements model of a wall.

With reference to the Lagrangian parameters \( \mathbf{u} = [u \ v \ \varphi] \)'s, the flexibility matrix of the MCP model can be written as

\[
\mathbf{f}^{\text{MCP}} = \begin{bmatrix}
\frac{1}{K_h} + \frac{1}{\lambda^2} \cdot \frac{c^2 h^2}{2\kappa \cdot K_v} & 0 & -\frac{1}{\lambda^2} \cdot \frac{c h}{2\kappa \cdot K_v} \\
0 & 0 & 1 \\
-\frac{1}{\lambda^2} \cdot \frac{c h}{2\kappa \cdot K_v} & 0 & 0
\end{bmatrix}
\]

(1)

where

\[
\kappa = \sum_{i=1}^{n} \left( \frac{1}{2} + n - i \right)^2 = \frac{n}{12} \left( 4n^2 - 1 \right)
\]

(2)

The elements of the flexibility matrix of the shell elements model can be easily obtained by applying unit forces to each degree-of-freedom and evaluating the corresponding displacements and rotations

\[
\mathbf{f}^{s} = \begin{bmatrix}
\hat{u}_F & 0 & -\hat{u}_M \\
0 & \hat{v}_N & 0 \\
-\hat{\varphi}_F & 0 & \hat{\varphi}_M
\end{bmatrix}
\]

(3)

By enforcing the equivalence between the two flexibility matrices the following expressions can be obtained:
These allow us to fully characterize the initial elastic behaviour of the discrete MCP model. In the present work the equivalence between the two models has been imposed in order to set the nonlinear model in the elastic range. However, with greater computational effort, a similar equivalence could also be enforced in the nonlinear range.

### 3.2 Nonlinear characteristics of the MCP model

For the nonlinear characterization of the MCP models both the properties of the vertical and the horizontal springs must be defined. The vertical springs have been modelled by considering the elasto-plastic behaviour of the steel in traction together with the nonlinear behaviour of the concrete in compression. The nonlinear behaviour of the horizontal spring has been defined considering a tri-linear force/displacement relationship, according to the model of Kabeyasawa et al. [5], represented in Fig. 7.

![Tri-linear force/displacement relationship proposed by Kabeyasawa.](image)

The stiffness of the initial branch, $K_h$, has been derived from the elastic equivalence with the shell model. The stiffness properties of the two successive branches, $K_h^{(2)}$ and $K_h^{(3)}$, have been evaluated as functions of $K_h$ according to the following empirical relationship:
\[ K_h^{(2)} = \left( 0.14 + 0.46 \cdot \frac{\rho_{wh} \cdot f_{wh}}{f'_c} \right) \cdot K_h^{(3)} \]  

(5)

\[ K_h^{(3)} = 0.001 \cdot K_h^{(1)} \]

where: \( \chi \) is the following shear factor

\[ \chi = \frac{3(1+\alpha) \cdot [1-\alpha'(1-\beta)]}{4 \cdot [1-\alpha'(1-\beta)]} \]  

(6)

\( G \) is the shear modulus; \( h \) is the inter-storey height; \( \rho_{wh} \) is the geometrical reinforcement ratio for the horizontal steel bars in the central panel with reference to the effective rectangular cross section, \( b_e \cdot L \), as reported in Fig. 8. \( f_{wh} \) is the yielding stress of the horizontal reinforcement steel; \( f'_c \) is the compressive cylindrical strength for the concrete; \( \alpha \) and \( \beta \) are geometrical parameters, as indicated in Fig. 8, \( \alpha = 1/L \), \( \beta = t/b \).

The value of the shear force at which the first crack in the concrete opens, \( V_c \), can be determined by means of the empirical formula

\[ V_c = 0.438 \cdot A_w \cdot \sqrt{f'_c} \]  

(7)

where the units of measurement must be: Newtons for \( V_c \), square millimetres for \( A_w \) and megapascals for \( f'_c \).

The value of the shear force at which the steel bars begin to yield \( V_y \), can be obtained as a function of the ultimate shear force \( V_u \) given by the following empirical expression due to Hirosawa [9]:

\[ V_u = \frac{7}{8} b_e \left( L - \frac{a}{2} \right) \left[ \frac{0.0679(f'_c + 17.6)\rho_t^{0.23}}{\sqrt{0.12 + \xi}} + 0.845 \sqrt{f_{wh} \cdot \rho_{wh} + 0.1 \cdot \sigma_0} \right] \]  

(8)

where \( L \), \( b_e \) and \( a \) are geometrical parameters defined in Fig. 8, that have to be expressed in millimetres; \( f'_c \) and \( f_{wh} \) must be expressed in megapascals; \( \rho_t \) is the ratio between the longitudinal steel rebars area in the border element in traction \( A_s \) and the effective cross section area \( (L-a/2) \cdot b_e \), expressed in percent:

\[ \rho_t = \frac{A_s}{(L-a/2) \cdot b_e} \cdot 100 \]  

(9)
Figure 8: Cross section of the wall considered in the TVLE model. (a) Geometry; (b) equivalent cross section; (c) shear area.

\( \sigma_0 \) is the average compression stress referred to the base section of the wall, that has to be expressed in megapascals; \( \xi \) is the ratio between the expected height of the point of application of the horizontal resultant force on the wall and the width of the wall; the ultimate shear force \( V_u \) is obtained in Newtons. Other authors proposed more complicated force-displacement relationships. These laws allow a more accurate simulation of the shearing response of the wall. Amongst these, one of the more relevant is the relationship proposed by Ozcebe and Saatchioglu in 1989 [10].

4 Finite element modelling

In order to investigate more thoroughly the seismic behaviour of the retrofitted building, and to judge separately the effects of the strengthening of the superstructure and of the base-isolation system, besides the model of the retrofitted base-isolated building, a model of the building with a fixed base the only strengthening of the superstructure has also been analysed.

4.1 The model of the fixed-base building

In the nonlinear 3-D model of the building on a fixed base the reinforced concrete walls have been modelled by means of MCP elements. Therefore the elastic characteristics of the MCP elements have been obtained for each wall,
according to the procedure described in the previous section, by stating the equivalence with the corresponding shell elements model. In the elastic range the accuracy of the overall 3-D finite element scheme, in which all the walls have been schematized as an MCP element (MCP-model), has been verified through a comparison with another model in which the walls have been modelled by means of 2-D shell elements (S-model). In Fig. 9 a comparison between the first three modes of vibration of both the models is reported together with the corresponding periods of vibration. A very good agreement between the modal shapes and the periods of the two models may be recognized.

4.2 The model of the base-isolated building

In the nonlinear model of the retrofitted base-isolated building the isolators have been introduced by means of nonlinear elastic springs. In effect each isolator has been schematized by means of a piecewise linear force/displacement relationship based on the characteristics provided by the manufacturer of the devices.

<table>
<thead>
<tr>
<th>TRANSVERSE</th>
<th>TORSIONAL</th>
<th>LONGITUDINAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-model</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$T = 0.163,s$</td>
<td>$T = 0.144,s$</td>
<td>$T = 0.156,s$</td>
</tr>
<tr>
<td>MCP-model</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$T = 0.166,s$</td>
<td>$T = 0.158,s$</td>
<td>$T = 0.154,s$</td>
</tr>
</tbody>
</table>

Figure 9: Comparison between the periods of vibration of the S-model and of the MCP-model of the building on a fixed base.

In Fig. 10 the first three periods, corresponding to the initial elastic range, of the base-isolated building evaluated considering either the model with the MCP elements or the model with shell elements are placed in comparison. As can be seen, the two models are in very good agreement.

The modes and the periods of vibration previously reported have been derived by considering the initial stiffness properties of the isolation devices. Since the
actual force/displacement relationship is nonlinear, the stiffness of the springs decreases as the lateral displacement increases, and therefore the periods of vibration of the elastic model actually depend on the level of deformation reached in the devices. Fig. 11 refers to the elastic design pseudo-acceleration spectrum provided by Eurocode 8 for A-type soil [11]. In this figure the fundamental periods on the longitudinal direction for the building on a fixed base with and without shear walls and the range of periods of the base-isolated structure are also reported. The two extreme values of this range \(T = 0.743 \div 1.46\) s correspond to the initial tangent stiffness and to the ultimate secant stiffness of the devices, since the superstructure remains in the elastic range. It is noteworthy that the base isolation provides a maximum reduction of the seismic action, with respect to the fixed base structure, equal to 27%.

<table>
<thead>
<tr>
<th>TRANSVERSE</th>
<th>LONGITUDINAL</th>
<th>TORSIONAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>(T = 0.750) s</td>
<td>(T = 0.751) s</td>
<td>(T = 0.431) s</td>
</tr>
</tbody>
</table>

\(S\)-model

\(MCP\)-model

\(T = 0.750\) s \(T = 0.743\) s \(T = 0.382\) s

Figure 10: Comparison between the periods of vibration of the \(S\)-model and of the \(MCP\)-model of the base-isolated building.

5 Pushover analyses

Pushover analyses have been performed both for the fixed-base building with shear walls and for the retrofitted building with shear walls and base isolation. Two force distributions, corresponding to the first two modes of vibration, have been considered for each model. These are reported in Fig. 12a for the fixed-base model and in Fig. 12b for the base-isolated model. In the case of the fixed-base building the force distributions are compared with the forces prescribed by
seismic codes, while for the base-isolated building a mass proportional distribution has been visualized for comparison. In the latter case the comparison highlights how both the considered modes of vibration are similar to a rigid translation of the superstructure.

Figure 11: Reduction of the seismic action for the base-isolated building according to the Eurocode 8 elastic design spectra for A-type soil.

Figure 12: Comparison between the forces considered in the pushover analyses: (a) fixed-base building; (b) base-isolated building.

The results of the pushover analyses have been expressed in terms of inter-storey drifts as functions of the base-shear coefficient. These are reported in Figs. 13 for the fixed-base building, and in Figs. 14 for the base-isolated building. Conventionally the analyses have been halted when a structural element has reached its ultimate state. This can be seen from the observation of
Fig. 14, in the case of the base-isolated building where the superstructure exhibits an elastic behaviour and the collapse condition is reached for $C_b=0.264$, for both the loading directions, when the ultimate displacement is attained in the isolation devices. This can also be observed in Fig. 15, where the lateral displacement of the isolation devices is reported as a function of the base-shear coefficient.

![Graph](image1.png)

**Figure 13:** Relative storey-drift as a function of the base-shear coefficient for the fixed-base building: (a) transverse direction; (b) longitudinal direction.

![Graph](image2.png)

**Figure 14:** Relative storey-drift as a function of the base-shear coefficient for the base-isolated building: (a) transverse direction; (b) longitudinal direction.

The values of the base-shear coefficient that bring the different models of the building to collapse under incremental static loading are reported in Table 2. In the table the values relative to the existing building (analysed in the companion paper [2]) are also reported for comparison.

The comparison highlights that for the base-isolated building, in both the loading directions, the collapse condition under monotonic static loads is reached
for values of the base-shear coefficient smaller than those of the same building on a fixed base. However, this result should not be surprising, because it is relative to an incremental static analysis and does not take into account the main effect of the isolation system that is associated to the strong modification of the dynamic characteristics of the structure with respect to the same structure on a fixed base.

However these obtained results will be useful in defining simplified equivalent systems that allow one to perform nonlinear dynamic analyses at a very low computation cost.

![Graph showing lateral displacement of isolation devices](image)

**Figure 15:** Lateral displacement of the isolation devices normalized with respect to the ultimate value as a function of the base-shear coefficient.

<table>
<thead>
<tr>
<th></th>
<th>Transverse direction</th>
<th>Longitudinal direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing building</td>
<td>0.088</td>
<td>0.160</td>
</tr>
<tr>
<td>Fixed-base building with shear walls</td>
<td>0.660</td>
<td>0.632</td>
</tr>
<tr>
<td>Retrofitted building</td>
<td>0.264</td>
<td>0.264</td>
</tr>
</tbody>
</table>

**Table 2:** Collapse base-shear coefficients.

6 **Simplified equivalent systems**

The procedure described in the companion paper [2] can be applied for the definition of simplified systems equivalent to the building on a fixed base. But the same procedure is not suitable for the definition of simplified systems equivalent to the base-isolated building, because a single degree-of-freedom is not sufficient to describe properly the behaviour of a base-isolated structure.
In order to define a simplified system equivalent to a base-isolated structure it is necessary to consider, at least, a degree-of-freedom for describing the superstructure’s behaviour (superstructure’s dof) and a further degree-of-freedom associated to the motion of the base (foundation’s dof). The nonlinear force/displacement relationship for the superstructure’s dof can be defined, through the same procedure described in the companion paper [2] for the single-degree-of-freedom system, by means of the results of the pushover analyses of the base-isolated building. The nonlinear characteristic of the base-isolation system can simply be taken into account by considering the overall nonlinear behaviour of the isolator devices according to the corresponding damping and stiffness properties. The main characteristics of the obtained simplified systems equivalent to the building on a fixed base and to the base-isolated building are reported in Table 3.

7 Seismic resistance

In what follows the results of the evaluation of the seismic resistance of the retrofitted building, according to the two procedures described in the companion paper [2], are presented. The first procedure is simply based on the use of design spectra, while the second procedure is based on the results of many nonlinear dynamic analyses performed on nonlinear equivalent systems. The seismic resistance will be expressed in terms of the maximum peak ground acceleration that the building can withstand according to a prescribed response spectrum (collapse-PGA). The peak ground acceleration that causes the collapse of the
building will be estimated both for the building on a fixed base and for the base-isolated building.

Table 3: Main results of the definition of the equivalent systems.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Transverse</th>
<th>Longitudinal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yielding $C_b$ for the bilinear system, $C_{b,y}$</td>
<td>0.549</td>
<td>0.591</td>
</tr>
<tr>
<td>Effective yielding base-shear coefficient, $C_{b,y'}$</td>
<td>0.547</td>
<td>0.554</td>
</tr>
<tr>
<td>Collapse base-shear coefficient, $C_{b,c}$</td>
<td>0.623</td>
<td>0.660</td>
</tr>
<tr>
<td>Effective yielding displacement, $u_y$ (cm)</td>
<td>0.57</td>
<td>0.48</td>
</tr>
<tr>
<td>Collapse displacement, $u_c$ (cm)</td>
<td>0.68</td>
<td>0.79</td>
</tr>
<tr>
<td>Effective period, $T_{eff}$ (s)</td>
<td>0.20</td>
<td>0.18</td>
</tr>
<tr>
<td>Hardening force ratio, $h = \frac{C_{b,c}}{C_{b,y}}$</td>
<td>1.14</td>
<td>1.1</td>
</tr>
<tr>
<td>Global ductility, $\mu$</td>
<td>1.05</td>
<td>1.48</td>
</tr>
<tr>
<td>Superstructure’s dof collapse base-shear, $C_{b,c}$</td>
<td>0.264</td>
<td>0.264</td>
</tr>
<tr>
<td>Superstructure’s dof collapse displacement, $\delta u_c$ (cm)</td>
<td>0.114</td>
<td>0.129</td>
</tr>
<tr>
<td>Superstructure’s dof effective period, $T$ (s)</td>
<td>0.132</td>
<td>0.140</td>
</tr>
<tr>
<td>Superstructure’s dof global ductility, $\mu$</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Foundation’s dof collapse lateral force, $F_{f,c}$ (kN)</td>
<td>3939.0</td>
<td></td>
</tr>
<tr>
<td>Foundation’s dof collapse displacement, $u_{f,c}$ (cm)</td>
<td>14.1</td>
<td></td>
</tr>
<tr>
<td>Foundation’s dof initial period, $T_{f,o}$ (s)</td>
<td>0.729</td>
<td></td>
</tr>
<tr>
<td>Foundation’s dof collapse secant period, $T_{f,sec,c}$ (s)</td>
<td>1.457</td>
<td></td>
</tr>
</tbody>
</table>

As described in [2], the evaluation of the seismic resistance by means of the design spectra approach is based on the knowledge of the effective period of the equivalent system and of the behaviour factor. However, for the base-isolated building, the fundamental period of the equivalent system is dependent on the level of deformation. Therefore, in order to apply this procedure for the base-isolated system, reference values of the period and of the behaviour factor must be chosen. Since the collapse condition is analysed, the period associated to the ultimate secant stiffness, corresponding to the ultimate displacement of the isolators, appears to be a suitable value for the application of the procedure. Furthermore, considering that the pushover analyses seem to indicate that the superstructure behaves elastically up to the collapse condition, for the evaluation of the spectral value reference will be made to elastic design spectra. For what concerns the viscous damping of the isolation devices, that is dependent on the lateral displacement, this can be set equal to a reasonable medium value of 10%.

The results of the procedure based on design spectra are reported in the following table together with the values relative to the existing building derived in the companion paper [2]. It is worth noting that the values of the collapse-PGA of the base-isolated structure are the same in both directions, this is because both in the longitudinal and transverse directions the system is
characterized by the same fundamental reference period and the collapse is associated to the ultimate condition for the isolators.

Table 4: Collapse-PGA according to the EC8 inelastic design spectra.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Transverse direction</th>
<th>Longitudinal direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing building</td>
<td>A</td>
<td>0.150 g</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.100 g</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>0.083 g</td>
</tr>
<tr>
<td>Fixed-base building with shear walls</td>
<td>A</td>
<td>0.261 g</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.261 g</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>0.290 g</td>
</tr>
<tr>
<td>Retrofitted building</td>
<td>A</td>
<td>0.472 g</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>0.315 g</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>0.262 g</td>
</tr>
</tbody>
</table>

Alternatively, the collapse-PGA can be estimated through nonlinear dynamic analyses performed on the simplified equivalent systems, according to the procedure described in the companion paper [2].

In order to perform the nonlinear dynamic analyses, cyclic behaviour must be defined for the equivalent system. For the fixed-base system the results of the pushover analyses had shown that the building reaches the collapse condition because the limit shear strength is attained in a shear wall. Since the shearing behaviour of the walls is characterized by the tri-linear relationship with origin-oriented unloading proposed by Kabeyasawa et al. [5], a single-degree-of-freedom system with origin-oriented unloading has been considered in the analyses. The evaluation of the collapse-PGA by means of the procedure based on the nonlinear dynamic analyses performed on the equivalent systems yields the results reported in Fig. 17a for the transverse direction, and in Fig. 17b for the longitudinal direction. In the same figures the values of the collapse-PGA obtained by the application of the first procedure are reported for comparison.

With reference to the fixed-base building with shear walls, it can be seen that for excitation acting in the transverse direction the results are in good agreement with those of the design spectrum procedure. When the excitation acts along the longitudinal direction the design spectrum procedure seems to overestimate the collapse-PGA. This could be realistically due to the fact that the energy dissipated in the single-degree-of-freedom equivalent system with origin-oriented unloading is smaller than that of an elasto-plastic system. Therefore the definition of the behaviour factor should be modified to account for this. The comparison between the results obtained with reference to the longitudinal direction for the fixed-base building with shear walls and for the existing building shows that, the collapse base-shear coefficient increasing by four times, the building with shear walls exhibits an average collapse-PGA smaller than the one relative to the existing building. This is due to the fact that the shear walls, even though increasing the resistance in terms of the collapse base-shear
coefficient, also produce an increment of the stiffness and therefore of the seismic action, particularly for firm soils.

Table 5: Collapse-PGA for the EC8 A-type soil design spectrum, evaluated by means of nonlinear dynamic analyses.

<table>
<thead>
<tr>
<th></th>
<th>Transverse direction</th>
<th>Longitudinal direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Existing building</td>
<td>0.213±0.022 g</td>
<td>0.334±0.030 g</td>
</tr>
<tr>
<td>Fixed-base building</td>
<td>0.255±0.007 g</td>
<td>0.310±0.008 g</td>
</tr>
<tr>
<td>with shear walls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Retrofitted building</td>
<td>0.494±0.027 g</td>
<td>0.493±0.025 g</td>
</tr>
</tbody>
</table>

Figure 17: Collapse-PGA for the equivalent systems. A-type soil, excitation along (a) the transverse direction and (b) the longitudinal direction.

The base-isolated building with shear walls exhibits values of the collapse-PGA in very good agreement with the results obtained by means of the simpler first procedure. The comparison between the base-isolated building and the building on a fixed base shows that the base-isolation produces an average increment of the collapse-PGA equal to about 90% in the transverse direction and to about 60% in the longitudinal, not to mention that the base-isolated system behaves elastically until the collapse condition, which is associated to the ultimate displacement of the isolators.
8 Conclusions

In this paper the seismic resistance of a retrofitted building, with reference to an expected earthquake action, has been estimated by means of two procedures based on the results of pushover analyses on 3-D finite element models as well as dynamic analyses on simplified equivalent systems. The nonlinear behaviour of the shear walls has been modelled according to discrete element models while the isolator devices have been introduced by means of nonlinear spring-dashpot elements. A procedure for the definition of a two-degrees-of-freedom system equivalent to a base-isolated building has been presented. This simplified system allows for an accurate modelling of the nonlinear stiffness and damping properties of the isolator devices, permitting one to perform a wide number of nonlinear dynamical analyses that produce a reliable estimation of the seismic resistance of the retrofitted building with a strongly reduced computational effort. The seismic performance of the retrofitted building, that has been conventionally expressed in terms of the minimum PGA that causes the building’s collapse according to a prescribed design spectral shape, has been evaluated. The obtained results show a very good performance of the isolated structure that is able to withstand the expected earthquake at the construction site without suffering any damage.

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References


